CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JUNE, 1954. **Vol. XLIX, No. 6**

FORTY-NINTH YEAR OF PUBLICATION

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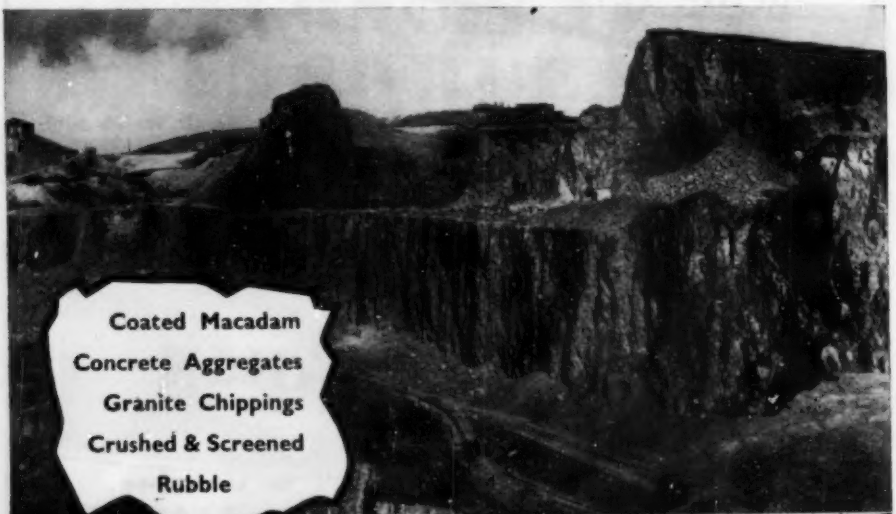

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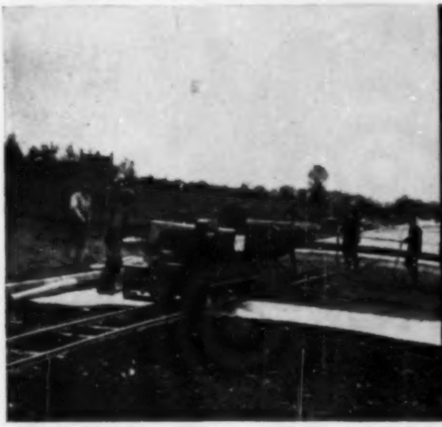

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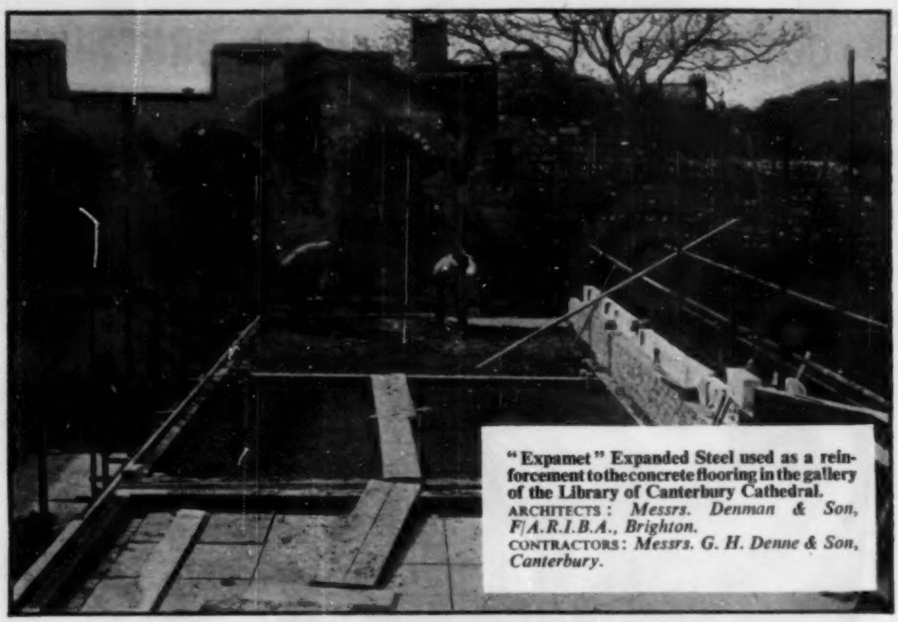

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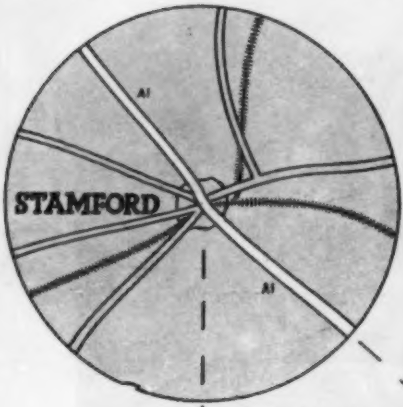

ANOTHER ANGLIAN PRESTRESSED PRODUCT

Prestressed concrete beams in use on the partly-completed road-over-rail bridge on the Great North Road at Stamford. This illustration shows one of several trunk road bridges at present under construction using "ANGLIAN" prestressed beams.

Photograph by courtesy of British Railways, Eastern Region. Contractors: Messrs. Wellerman Bros. Ltd., Sheffield

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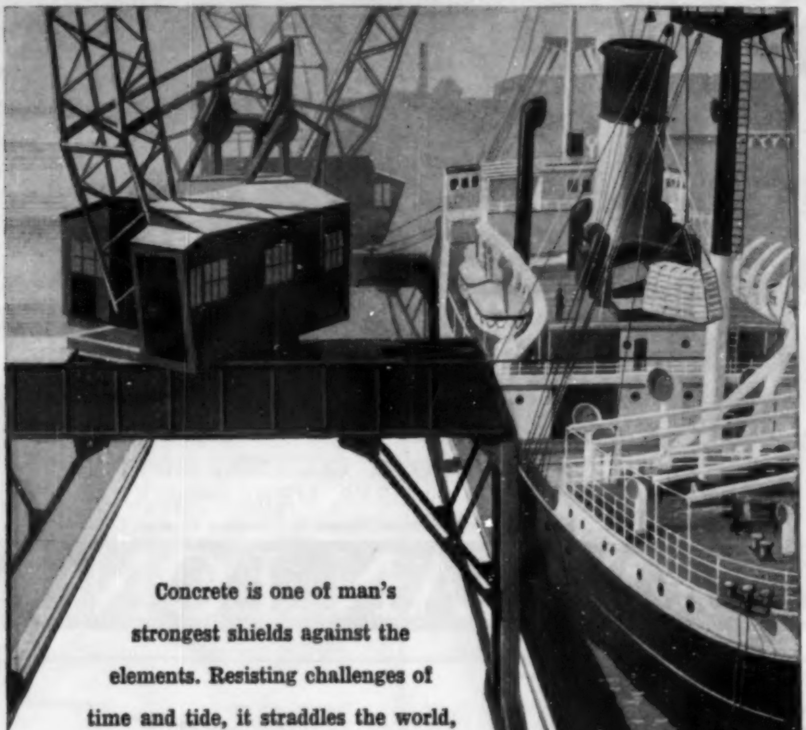

CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume XLIX, No. 6. LONDON, JUNE, 1954.

EDITORIAL NOTES

The Quality of Concrete.

IN recent years much attention has been given to the production of concrete of uniform strength. The useful results of all this research have been small. Most investigators have concentrated on refinements of present methods of making concrete, and few attempts seem to have been made to eliminate some of the variables which occur between the quarry and the hardened concrete. These variables in the raw materials, the proportioning, the water content, the mixing, the unmixing between the mixer and the shuttering, the lack of consolidation or the further separation due to careless spading or vibrating, the curing process, and so on, are the reasons why we must have concrete with an average compressive strength of 3000 lb. per square inch, and a maximum very much higher, before it is safe to stress it to 1000 lb. per square inch. The elimination of some of the reasons why the permissible compressive strength can be such a small proportion of the average strength is much more important than mathematical exercises such as the statistical analyses of test results and suggestions that risks be taken by ignoring the lowest.

How little we have progressed in producing concrete of more uniform strength is shown in some of the papers read at a symposium on the design of concrete mixtures and the control of the quality of concrete held in London last month under the auspices of the Cement and Concrete Association. The suggestion that the problem of relating the required average compressive strength to a specified minimum strength requires a statistical approach, and that the engineer must decide what risk he will take of a cube strength being below the minimum, seems to be a policy of despair and a risky one at that. For example, in a dam now in course of construction a thousand cubic yards of concrete are being placed each day, and three test cubes are being made at intervals during the day of eight hours. The cube with the lowest strength might be merely a badly-made specimen and not at all representative of the concrete in the work, it might be that only one mixer-batch was of poor quality, or it might be that one-third of the concrete placed during the day, that is 330 cu. yd., was of inferior quality. In one paper a table is given showing that the deviation from the average strength of fifty specimens made under closely-controlled conditions was 12·2 per cent. when calculated as "standard deviation", but that the strength of three of the cubes was 23 per cent. below the average; this represents 6 per cent. of the concrete, and it seems risky to ignore so much concrete having a strength so much below

the so-called "standard deviation," and which may be in a part of the structure subjected to the greatest stress. In another paper it is suggested that in the case of concrete with an average strength up to 3000 lb. per square inch there is no need to make any investigation so long as the strength does not vary by more than 30 per cent. to 80 per cent. from the average, according to the amount of supervision. In the case of high-quality concrete with an average strength above 3000 lb. per square inch it is stated that investigation is advisable only if the strength varies by 1000 lb. per square inch from the average, which seems to mean that if the average is 3100 lb. per square inch a variation between 4100 lb. and 2100 lb. per square inch must be considered as normal. No doubt such variations are common experience, and this is the reason why it is safe to assume that concrete has a strength of perhaps about one-quarter of the strongest, one-third of the average, and one-half of the lowest test results, and an unknown proportion of the unknown strength of the concrete when it is in the structure.

It is doubtful if such variations of the strength of the concrete in one structure were greater twenty years ago than they are to-day. Refinements in existing methods of securing higher strengths and greater uniformity have been introduced in recent years, but the concrete in one part of a structure may still be less than half as strong as in other parts. Something more than new methods of interpreting tests results and refinements by laboratory workers is necessary to remedy or seriously improve this state of affairs. Methods of distributing concrete without barrows or chutes, of removing excess water before the concrete hardens, and of cementing together ungraded coarse aggregate with grout, are among the newer radical departures from the traditional methods of concreting, and it seems that only by radical new methods will greater uniformity be obtained. Experiments have shown that it is possible to make a cement that is proof against moisture and can be stored in the open without deterioration. Is it not much more worth while to try to produce a concrete that will set and harden without the addition of large quantities of water? Such a concrete would solve most of the problems, for most of the troubles of concrete are due to the presence of more water than is necessary for the chemical reaction of the cement.

The author of one of the papers has gone a useful step farther. He has discovered that, for the same proportion of cement to combined aggregate, the strength of the concrete is the same whatever the grading so long as the total surface area of the particles of aggregate in a given volume is the same. In the tests the aggregate was separated into seven different sizes, the largest passing a ¾-in. sieve and the smallest retained on a No. 100 sieve. The first mixture contained aggregate of all seven sizes, the second material of the three smallest and two largest sizes, the third the two smallest and two largest sizes, and the last the smallest and largest sizes only. The average compressive strength of all these concretes was 3475 lb. per square inch at seven days with a greatest variation of 95 lb. per square inch; at 28 days the average strength was 4717 lb. per square inch and the great variation only 67 lb. per square inch. This discovery should be useful in districts where graded aggregates are difficult to obtain.

An important step forward are courses for supervisors of concrete starting this year at the City and Guilds of London Institute, and at Croydon, Cardiff, and Edinburgh, on the suggestion of the Reinforced Concrete Association. A list of papers presented to the symposium is on page 206.

Reinforced Concrete Arch Bridge in Brazil.

By E. HAROLD SIDWELL.

A BRIDGE (*Fig.* 1) across the Rio das Antas in the south of Brazil, which is believed to be the second largest reinforced concrete arch in the world, has recently been completed. The arch has a span of 180 metres (610 ft.); the longest arch in the world is Sandö bridge in Sweden, which has a span of 866 ft. The river is subject to very rapid rises following rain. One set of wooden falsework was swept away by flood, and was replaced up to deck level by lattice steel pillars supporting lattice girders on which was erected the wooden falsework for the arch ribs and the shuttering for the deck (*Figs.* 2 and 3). The steelwork was designed so that it could be dismantled and used for another bridge on a nearby site.

Specification NB—6 for road bridges and the loading for Class 1 roads and NB—2 for the calculation and construction of reinforced concrete bridges (see Appendix 1) were used in the design. The principal traffic is heavy lorries carrying

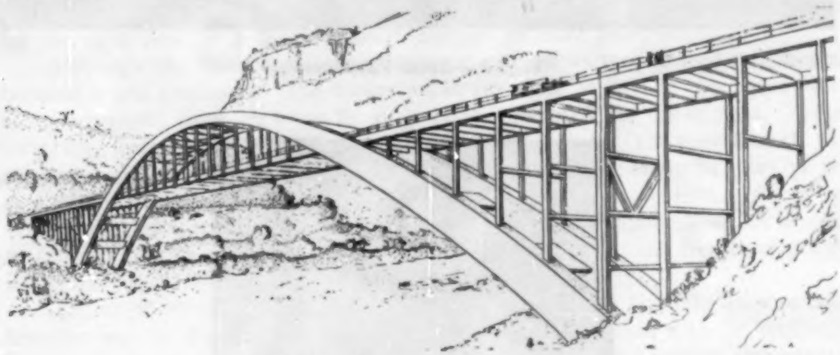


Fig. 1.—Arch of 610 ft. Span.

timber from pine forests in the neighbourhood. The two arch ribs are hollow in cross section, varying from 5 m. (16·4 ft.) in depth at the springings to 3 m. (9·84 ft.) at the crown. Their width is constant at 1·5 m. (4·93 ft.), and their centres are 8·7 m. (28·5 ft.) apart. A span of 180 m. (610 ft.) and a rise of 28 m. (92 ft.) above the springings give a rise-to-span ratio of 1 to 6·4. The ribs are of catenary shape and carry a dead load (their own weight and that of the deck and supporting members) of 4·2 tons per foot at the crown and 6·95 tons per foot at the springings. The supporting posts are 23·6 in. square and the horizontal bracing between the arch ribs is of K shape above the deck and in the form of a slab below the deck. Contraction joints are provided at the middle of the length of the deck and above the springings, where the supporting columns are duplicated and separated by 1·18 in. The ribs were designed for a temperature variation of 5 deg. C. from the normal, the bridge being at a latitude of about 30 deg. south. Shrinkage of the concrete was assumed to be equivalent to a fall in temperature of 15 deg. C.; the coefficient of contraction of reinforced concrete in Brazilian specifications is 10^{-5} per degree C. The viaducts at each end of the bridge consist of a series of in-situ frames as shown in *Fig.* 4. The

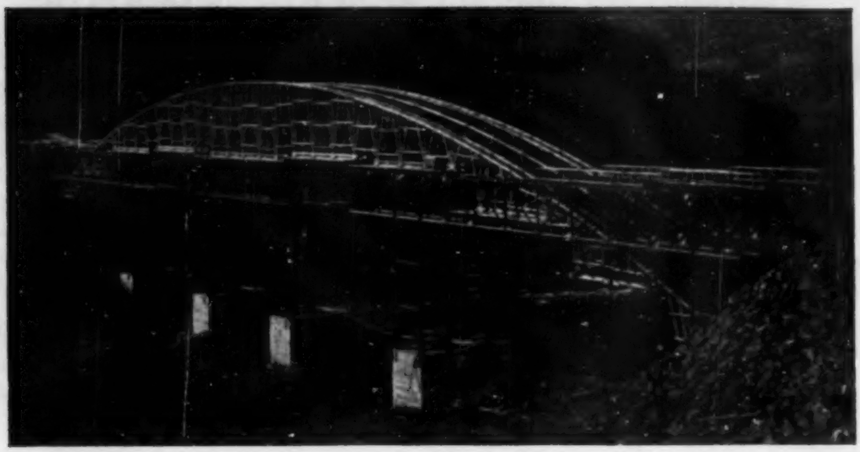


Fig. 2.—Steel Falsework.

Fig. 3.—Steel Falsework.

total length of the bridge and viaducts is 915 ft. The viaducts have slender members to produce structural stability with a pleasing appearance.

Economy in materials is very important in Brazil, and the details of a design are considered much more than in Britain. Concrete sections are probably more slender than anywhere else in the world. The calculations for the viaduct frames were compared with experiments on celluloid models at the Instituto Tecnologia at Porto Alegre. After construction, strains in the pillars were measured with Huggenberger gauges while the trestle was subjected to a lateral load. Agreement among the different methods was reported to be good; none of these, however, revealed an interesting vibration problem which occurred when the viaducts and their decks were completed. By standing and twisting one's body on the deck of the viaduct at the contraction joint one could easily cause oscillations of as much as ⅜ in.; this was rectified by the provision of inverted V-shaped members as shown in *Figs.* 1 and 4. A crack occurred in one of the edge-beams supporting the deck of the viaduct. Opinions differed on whether the slight settlement of the foundation which caused this was due to occasional layers of soft decomposed rock in the excavation, or to the use of dynamite in excavating the foundations for the arch ribs; the latter opinion seemed to have more support.

Although the local aggregate was not as cubical as was desired it was used because it was available. The concrete had strengths of 3880 lb. per square inch at twenty-eight days, and 640 lb. of cement were used per cubic yard. Other quantities were—In the arch ribs: 2420 cu. yd. of concrete; 128 tons of steel. In the deck: 655 cu. yd. of concrete; 108 tons of steel. In the viaducts: 523 cu. yd. of concrete; 49·2 tons of steel. In the foundations of the arches: 1070 cu. yd. of concrete; 32·4 tons of steel. The cost was 16,147,000 cruzeiros, which is equal to £107,500 at the present rate of exchange or £215,000 at the free market rate when the bridge was commenced.

The engineer of the bridge was Dr. Antonio Alves de Noronha (whose chief designer was Sr. Leopoldo de Castro Moreira) in co-operation with Sr. Anthonio Froes and Sr. Solon Fonseca, Chief Engineer and Bridge Engineer respectively of Rio Grande do Sul—the State in which the bridge is built. The contractors were Messrs. Christiani & Nielsen. The writer was privileged to work in the office of Dr. Noronha at Rio de Janeiro for nearly a year.

Appendix I.

Brazilian Code NB—6 of 1950 specifies the live loads to be considered in the calculation of three classes of road bridges. The loads are composed of road rollers, lorries, and miscellaneous traffic. Details of the vehicles are given in *Tables* I and II.

Table I.—Road Rollers.

	Type A	Type B	Type C
Total weight (tons)	6·88	15·72	23·60
Weight (tons) of front roller	4·92	6·98	9·84
Weight (tons) of each rear wheel	0·98	4·42	6·88
Width of front roller (ft.)	3·28	3·28	3·28
Width of each rear wheel (in.)	3·94	15·75	19·68
Distance between front and rear axles (ft.)	9·84	9·84	9·84
Distance between centres of rear wheels (in.)	62·99	62·99	62·99

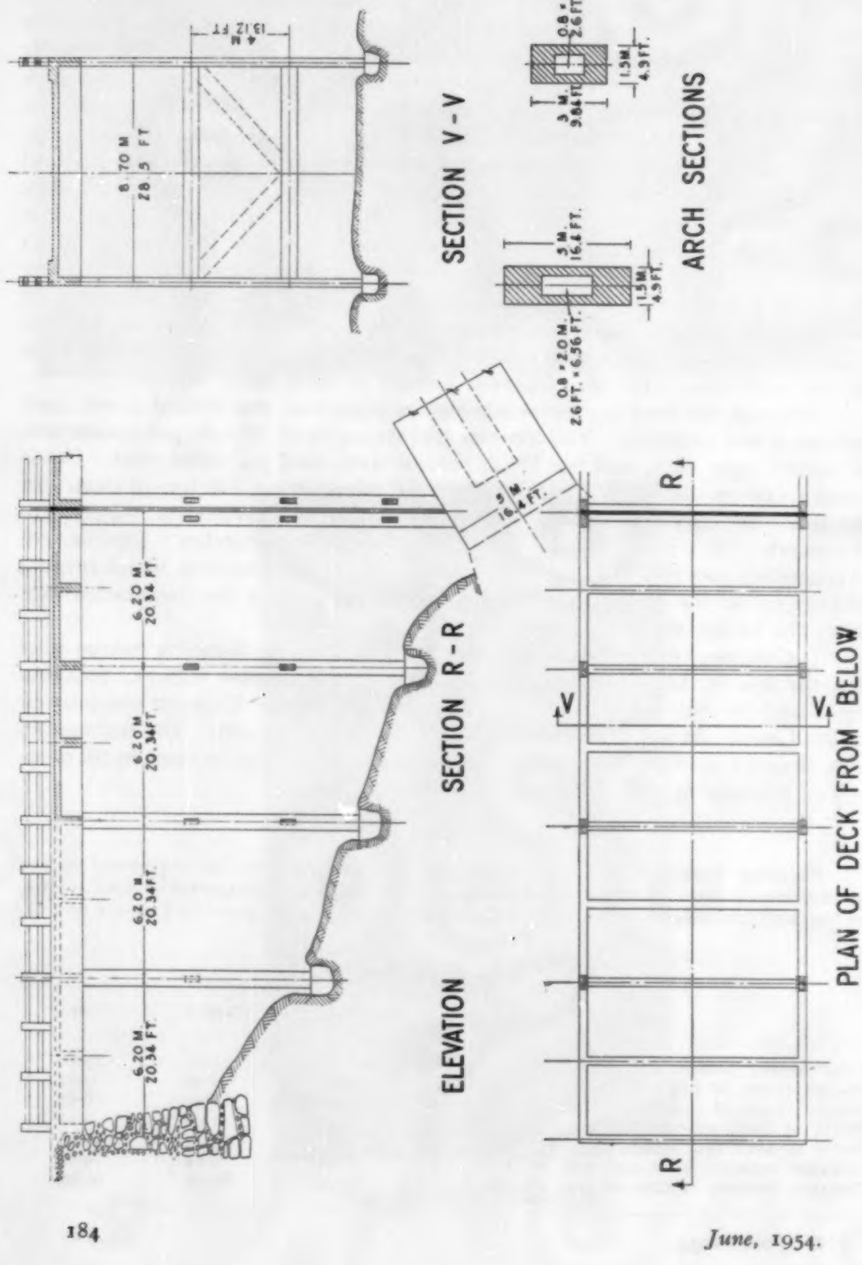


Fig. 4.—Details of Viaduct.

TABLE II.—LORRIES.

	Type A	Type B	Type C
Total weight (tons)	5·92	8·85	11·81
Weight on each front wheel (lb.)	1655	3310	4410
Weight on each rear wheel (lb.)	4970	6620	8820
Width of each front wheel (in.)	3·15	4·72	4·72
Width of each rear wheel (in.)	7·08	9·45	9·45
Distance between front and rear axles (ft.)	9·84	9·84	9·84
Distance between centres of rear wheels (in.)	62·99	62·99	62·99

The live load due to miscellaneous traffic is to be calculated as a uniformly-distributed load whose intensity is given as a function of the parameter g_0. Arches or main beams are calculated as follows.

	kg. per sq. m.	lb. per sq. ft.
(a) With less than 25 m. (82 ft.) of theoretical span	g_0	g_0
(b) With span l between 25 m. and 125 m. (82 ft. and 410 ft.)	$[g_0 - (l - 25)]$	$\left[4{\cdot}88g_0 - \left(\frac{l}{3{\cdot}28} - 25\right)\right]0{\cdot}205$
(c) With more than 125 m. (410 ft.) of theoretical span	$g_0 - 100$	$[4{\cdot}88g_0 - 100]0{\cdot}205$
(d) For the calculation of the other members	g_0	g_0

It is permitted to ignore fractions of 10 kg. per square metre (2 lb. per square foot).

Miscellaneous traffic is assumed to be distributed on the footpaths and the part of road not occupied by vehicles. For this purpose the area occupied by a vehicle (roller or lorry) is assumed to be a rectangle 8·2 ft. wide and 19·69 ft. long with the centre over the longitudinal axis of the vehicle midway between the front and rear axles.

In calculating arches or main beams it is permissible to ignore the effect of the redistribution of loads caused by the secondary beams. In the case of arches or main beams of 98·43 ft. or more span it is permissible to substitute concentrated loads of vehicles (roller or lorry) of equal load but uniformly distributed over a rectangle 8·2 ft. wide by 19·69 ft. long.

Class 3.—The loads for Class 3 bridges consist of $g_0 = 82$ lb. per square foot for miscellaneous traffic, a roller of type A, and as many lorries of type A as there are lines of traffic, less one, all pointing in the direction of the traffic, and in the most unfavourable position for the calculation of each member, with the exceptions that there must not be more than one vehicle in each traffic line, nor must there be less than 8·2 ft. between the longitudinal axes of two vehicles. The calculations must also allow for one roller of type B in the most unfavourable position for the member under consideration, but pointing in the direction of the traffic.

Class 2.—The loads comprise $g_0 = 92{\cdot}5$ lb. per square foot for miscellaneous traffic, a roller of type B, and as many lorries of type B as there are traffic lines, less one, placed as described for Class 3 bridges. The calculations must allow for a roller of type C placed as for Class 3 bridges.

Class 1.—The loads comprise $g_0 = 102{\cdot}5$ lb. per square foot for miscellaneous traffic, a roller of type C, and as many lorries of type C as there are traffic lines, less one, and placed as described for Class 3 bridges.

Brazilian Code NB—2 of 1950 specifies the vertical impact factors for road bridges as follows.

Elements of the roadway (slabs, cross beams, longitudinal beams, struts, ties), 1·3.
Straight main beams : Spans up to 65·7 ft., 1·3 ; Spans greater than 230 ft., 1·0.
Arches : Spans up to 164 ft., 1·2 ; Spans greater than 230 ft., 1·0.

For intermediate spans the coefficients of impact may be obtained by linear interpolation.

Vertical impact is not considered in the cases of (*a*) The transformation of loads for the calculation of earth pressures; (*b*) Bearings, solid pillars, and foundations; (*c*) Footpaths.

Precast Concrete Bridges in Australia.

In a recent report by the Engineer of the Country Roads Board of the State of Victoria, Australia, reference is made to the increasing use of precast concrete for road bridges. For spans between 4 ft. and 20 ft. U-type precast slabs are used, and for spans between 22 ft. 6 in. and 37 ft. 6 in. precast T-beams have been

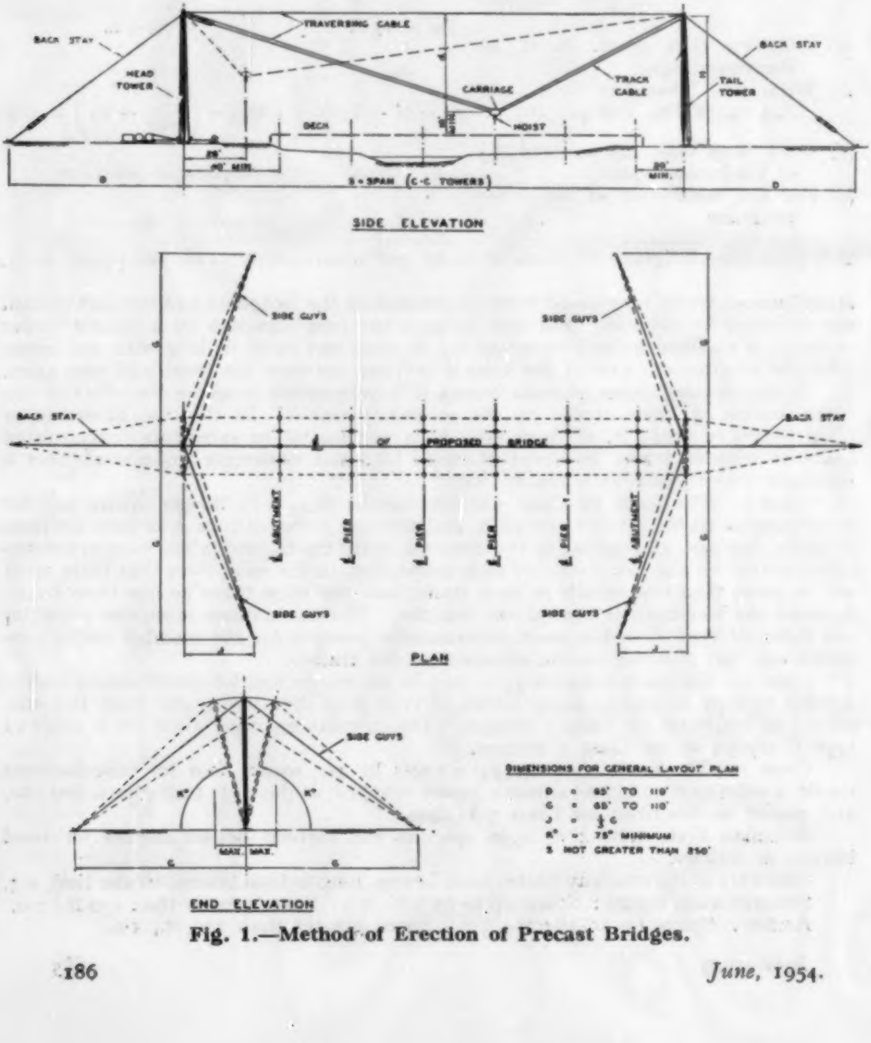


Fig. 1.—Method of Erection of Precast Bridges.

designed with precast deck slabs of the type shown in *Fig.* 2.

The erection of the larger bridges is by means of a cableway (*Fig.* 1) the towers of which are made up of sections 15 ft. long, each comprising four mild steel angles braced by steel bars welded to them. The heaviest weight to be lifted for any bridge is 4·3 tons for the cross-heads, and with the aid of the cableway three of these may be placed in four hours.

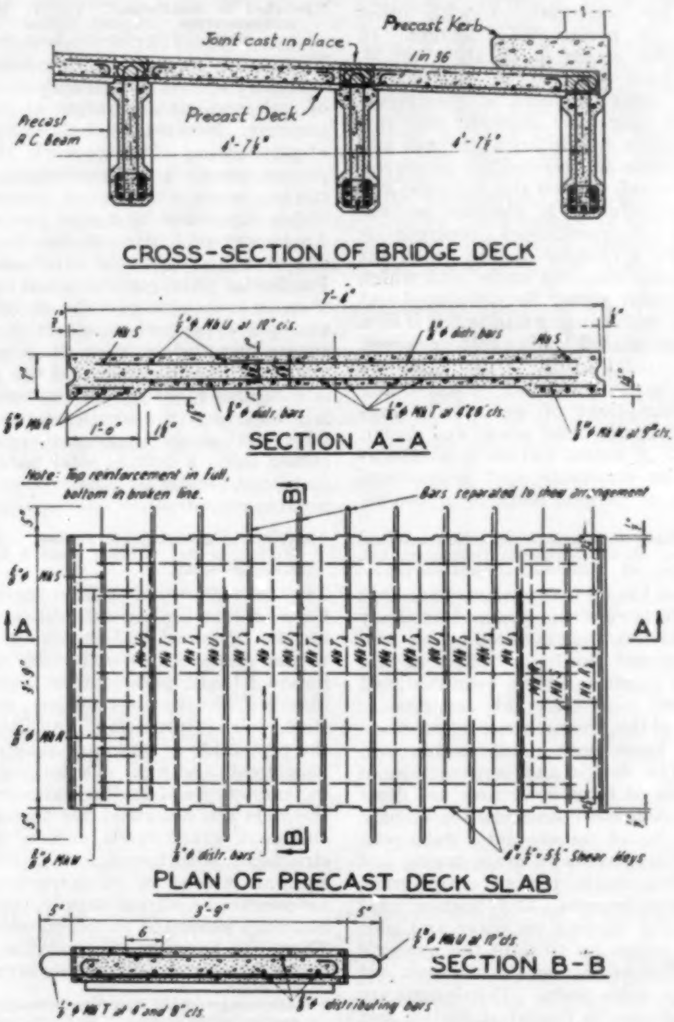


Fig. 2.—Details of Precast Concrete Bridges.

Book Reviews.

"Prestressed Concrete Design and Construction." By F. Walley. (H.M. Stationery Office. Price 30s.)

IN his preface the author states that this book is based on the experience in prestressed concrete of the Ministry of Works and on his conversations with other designers in prestressed concrete. In view of the recent publication of M. Guyon's book and of the existence of books of other pioneers of prestressed concrete it cannot be expected that this latest addition to the literature will add much to common knowledge. However, the book is well written and has good and clear illustrations. A chapter on the principles of prestressed concrete is followed by a chapter on the behaviour of prestressed concrete under load which includes tables giving the calculated and the actual loads causing failure; it is seen that the calculated loads agree very well with the actual loads. Other chapters deal with materials, losses of tensioning forces, descriptions of methods of prestressing, experimental work, and design problems. A useful feature is a tabulation of the constants and geometrical properties of various sections of beams.

"Concrete Farm Structures." By A. M. Pennington. 150 pages. (London: Concrete Publications, Ltd. 1954. Price 12s. 2.80 dollars in North America.)

THE author has for many years specialised in concrete farm structures and ancillary works, and this book is based on his own experience and methods. Most of the structures illustrated have been designed by him and built under his supervision, and most of the precast concrete products described have been made in his own works. The design and construction of most types of farm structures are dealt with, including cowhouses, dairies, stables, piggeries, liquid-manure pits, dung-pits, barns, implement sheds, grain drying and storage silos, roads, paving, cattle-grids, gates, water-troughs, and horticultural frames. The chapter on silage and silos for grass brings up to date the author's book on this subject which has been out of print for some years. The designs are in accordance with the latest British and American practice, and also with British Standards and the recommendations of the Ministry of Agriculture where these are applicable. Fully-detailed working drawings are given for all parts of the structures dealt with, and recommended methods of making the precast products are described at length. The last chapter is on the planning of farms.

"Brücken in Stahlbeton." Vol. I. Platten-und Balkenbrücken. By Carl Kersten. 8th edition. 1953. (Berlin: Wilhelm Ernst & Sohn. Price 19 D.M.)

THIS work contains all the information necessary for the design and construction of slab and girder bridges in reinforced concrete. New matter deals with methods of prestressing in bridge work, the use of precast members, and methods of ensuring rigidity when a reinforced concrete deck slab is supported by a steel girder bridge. An important feature of this book is the 725 clear illustrations of structural details. For bridge parapets which are important from an æsthetic aspect the author prefers the open-type steel construction so as to ensure that the elevation shall be as light and graceful as possible, and this principle is well illustrated by photographs. This is in contrast to the practice in this country where, primarily in order to protect the public using a bridge, solid parapets are employed (Waterloo Bridge, London, is a notable exception).

"Berechnungsgrundlagen für Bauten." By Bernhard Wedler. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 7.50 D.M.)

THE twenty-second edition includes the German standard specifications required for the design of buildings, data relating to the properties of materials, dead and imposed loads, pressures on foundations, masonry, timber construction, structural steel, sound insulation, ventilation and the protection of buildings against dampness, frost, and fire. Scaffolding, including many types of cradles, is also dealt with. Methods are described for the design of chimneys, grandstands, colliery headgear structures, steel towers for supporting the equipment used for oil extraction, masts for overhead electrical supply, cranes, and buildings subjected to mining subsidence. There is, however, no mention of the design of reinforced concrete structures.

"Verdichtungstechnik und Verdichtungsgeräte im ausländischen Erdbau." By Heinz Pösch. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 9.80 D.M.)

DESCRIBES the methods of soil compaction developed in France, Great Britain, and the U.S.A., and the plant employed for this purpose.

German Recommendations for Prestressed Concrete.

RECOMMENDATIONS for design and construction in prestressed concrete have been issued in Germany. The recommendations relate to complete prestressing with no tensile stress in the concrete and partial prestressing in which some tensile stress in the concrete is permitted. The required properties of the concrete and steel are given in *Table* I.

SECTIONAL AREAS. — In calculating stress due to load and the factor of safety against cracking, the area F_i and moment of inertia I_i are obtained by considering the section as a whole, reckoning the area of steel as $(m - 1)$ times its actual cross section. The area of steel must be sufficient to resist all the tensile forces. Poisson's ratio (the coefficient of transverse strain) may be assumed to be $\mu = \frac{1}{6}$.

CREEP AND SHRINKAGE.—Losses due to the creep of steel may be disregarded provided that the stress in the steel is less than the creep limit, or that strain due to creep is prevented. The relationship between deformation (ε) and constant stress (σ) is given in *Fig.* 1; it is assumed that the deformation of the concrete due to creep is proportional to the stress and that the modulus of elasticity of the concrete E_c is constant; ϕ and α_K vary with time and tend towards their limiting values of ϕ_∞ and α_∞ after about four years. The values of these coefficients are given in *Table* II.

These coefficients are related to the strength of the concrete at the time of application of the forces producing creep, and are taken from *Fig.* 2 and *Table* I for various ratios of $\frac{W}{W_\infty}$ which define the strength, where W is the cube strength at the time of transfer and W_∞ is the strength at 4 years; W_∞ is considered to vary from 1·15 to 1·30 W_{28} (W_{28} is the strength at 28 days). Prior to these recommendations, the decrease in the prestress was assumed to vary from 19,900 lb. to 21,400 lb. per square inch.

TABLE I.—PROPERTIES OF STEEL AND CONCRETE.

	Steel (E_s) (lb. per sq. in.)	Concrete (lb. per sq. in.)			Modular ratio
Cube strength at 28 days . .	—	4280	6410	8550	—
Minimum strength at "transfer"	—	3420	5140	6850	—
Modulus of elasticity (E_c) . .	—	4,280,000	4,980,000	5,700,000	—
Hot-rolled wires and bars . .	30,000,000	7·0	6·0	5·3	$\frac{E_s}{E_c}$
Cold-rolled or drawn wires and strips	28,400,000	6·7	5·7	5·0	,,
Stranded cables comprising 2 to 7 wires (cold drawn or rolled) having a pitch of not less than 10 times the diameter of the cable	25,600,000	6·0	5·1	4·5	,,
All other types of steel according to test results . . .	—	—	—	—	,,

Notes.—Welding of steel is prohibited. Tolerance of tensioning force ± 5 per cent.

TABLE II.—COEFFICIENTS OF CREEP AND SHRINKAGE.

Position	Creep (ϕ_∞)	Shrinkage ($\varepsilon_s \times 10^{-5}$)
In water . . .	0·5K to 1K	0
In moist air . .	1·5K to 2K	10
In open air generally	2K to 3K	20
In dry air . . .	2·5K to 4K	30

DESIGN CALCULATIONS.—The maximum tensile stress in the member occurs before the prestress is applied. As this is a temporary condition, higher stresses are permissible as indicated in line 22 of *Table* III. After the application of the prestress, and before the full dead and live loads are applied, the maximum compressive stress

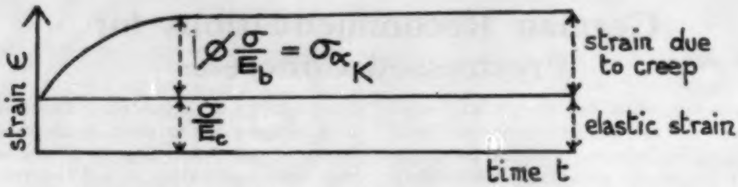


Fig. 1.—Relationship between Deformation and Constant Stress.

TABLE III.—PERMISSIBLE STRESSES.

Application	Strength of concrete at 28 days (lb. per square inch)		
	4280	6410	8550
CONCRETE.			
Compressive—			
In compressive area at design load*:			
1. Rectangular sections: Axial bending in one direction	1568	1990	2280
2. Tee-beams and hollow sections	1420	1850	2140
3. Direct compression	1140	1568	1850
In tensile area at design load:			
4. Rectangular sections: Axial bending in one direction	1990	2560	3000
5. Tee-beams and hollow sections	1850	2420	2840
6. Direct compression	1568	2060	2420
Tensile—			
Full prestress at design load:			
7. In compressive area before full loading*	428	541	641
In compressive and tensile areas after full loading:			
8. Generally	0	0	0
In some cases described in the code:			
9. Direct Tension	114	142	171
10. Axial bending in one direction	285	356	428
Partial prestress at design load:			
11. In compressive area before full loading	428	541	641
In compressive area at full load and in tensile area with reinforcement proportionately distributed:			
11a. Generally	0	0	0
12. Direct tension	171	214	256
13. Axial bending in one direction	428	541	641
In some cases described in the code:			
14. Direct tension	214	285	356
15. Axial bending in one direction	570	712	855
Shearing—			
Principal inclined tensile stresses at design load:			
16. Full prestress and shearing stress due to transverse force	114	128	142
17. Partial prestress and shearing stress	228	285	342
Principal inclined tensile stresses at breaking load:			
18. Shearing stress due to transverse force	455	570	685
19. In bending before shearing force is established	228	285	342
Bond—			
20. At design load	114	128	142
21. At breaking load	199	228	256
PRE-TENSIONED STEEL.			
22. During tensioning, $\leqq 0{\cdot}8\sigma_s$			
23. At design load, $\leqq 0{\cdot}75\sigma_s$. $\leqq 0{\cdot}55\sigma_B$			

Note.—σ_s = yield stress. σ_B = stress at failure. * The "design load" may be different at different stages in the construction. "Full load" includes live load, dead load, the effects of creep, shrinkage, etc.

occurs in the tensile area of the concrete and the maximum tensile stress in the compressive area. As this state is of short duration and is counteracted when the live load is applied, higher stresses may be used in accordance with lines 4 to 7 and 11 of *Table* III. The maximum stress often occurs before the commencement of creep and shrinkage. After creep and shrinkage have taken place there occurs the most unfavourable combination of stresses. This results in minimum compressive stress and maximum tensile stress in the tensile area of the concrete, maximum compressive stress in the compressive area, and, in some circumstances, in the maximum stress in the member. In most cases this combination produces also the maximum shearing stress.

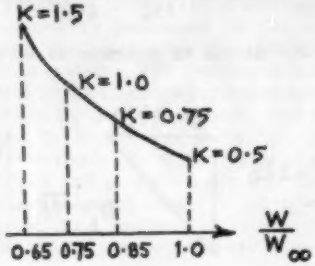


Fig. 2.—Coefficients of Creep and Shrinkage.

FACTORS OF SAFETY.—A factor of safety against cracking is mostly required. Under combined dead and live loads, tensile stresses in the tensile area of the concrete are not in general permitted.

TABLE IV.—PERMISSIBLE TENSILE STRESSES IN CONCRETE IN BRIDGES CARRYING NORMAL RAILWAY TRAFFIC.

Application	Strength of concrete at 28 days (lb. per square inch)		
	4280	6410	8550
At full prestress—			
1. In compressive area during construction	114	142	171
2. In compressive area with full dead load and in tensile area with all dead and live loads	0	0	0
In compressive area at full dead load and live loads and in tensile area with all dead and live loads and forces due to temperature variations, friction on bearings, etc.:			
3. Direct tension	114	142	171
4. In bending	214	285	356
Partial prestress—			
5. In compressive area during construction	114	142	171
With full dead load:			
6. In compressive area	0	0	0
In tensile area for all dead and live loads, with proportionate distribution of reinforcement:			
7. Direct tension	171	214	256
8. In bending in lower tensile area and in upper tensile area with sealing coat	356	428	498
In upper tensile area:			
9. With asphalt finish	285	356	428
10. Without asphalt finish	171	214	256
With full dead and live loads:			
11. In compressive area generally	171	214	256
In tensile area with all dead and live loads and forces due to temperature variations, friction on bearings, etc.:			
12. Direct tension	214	285	356
In bending:			
13. In lower tensile area generally and in upper tensile area with sealing coat	428	541	641
14. In upper tensile area with asphalt finish	356	428	541
15. Without asphalt finish	214	285	356

Certain exceptions are, however, allowed in bridges and other structures where additional or secondary loads are included (lines 3 and 4 of *Table* IV and lines 9 and 10 of *Table* III). Tensile stresses are permitted in the compressive area (lines 7, 9 and 10 of *Table* III and lines 1, 3, and 4 of *Table* IV). Calculation of the safe load must be based on the most unfavourable combination of loads comprising the prestressing force, creep, shrinkage, 1·75 times the sum of the dead and live loads, and temperature variations. A large factor of safety for concrete is required so that the working stress is limited to two-thirds of the stress causing failure.

MOMENT OF RESISTANCE.—To assess the total compressive strength in bending and the lever-arm of the internal forces for a section, a graphical method employing stress-strain curves is recommended. For rectangular or nearly rectangular sections the compressive force in the concrete may be assumed to be

$$D_c = 0{\cdot}75 \times \tfrac{2}{3} \times W_{28} \times F_{bd}$$
$$= 0{\cdot}5 \times W_{28}.F_{bd},$$

where F_{bd} is the sectional area of the concrete in compression, $\tfrac{2}{3}$ is the reduction coefficient, and 0·75 is the coefficient to determine the area below the stress-strain curve as in *Fig.* 3. The distance of the point of application of the force representing the total concrete in compression measured from the compressed edge is 0·4 times the depth of the compressive area in bending.

SHEARING AND BOND STRESSES.—The magnitude and direction of the principal tensile stresses are determined from the shearing and compressive stresses. Where only compressive stresses are developed, the principal tensile stress in any part of the section must be calculated to find its maximum value; this is the case of full prestress (*Fig.* 4). If the stress distribution indicates compression and tension, the operative maximum value of the principal tensile stress is at or above the neutral axis within the concrete in compression; this is the case of limited or partial prestress (*Fig.* 5). In design load calculations, the principal tensile stress must not exceed the values in line 16 for full prestress and line 17 for partial prestress (*Table* III). In calculations for breaking load, the principal tensile stress must not exceed the values in lines 18 and 19 of *Table* III.

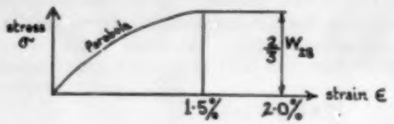


Fig. 3.—Stress in Relation to Strain.

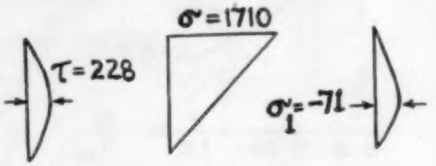


Fig. 4.—Principal Stresses (Compression only).

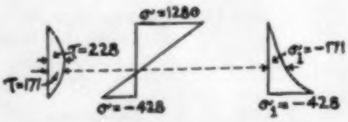


Fig. 5.—Principal Stresses (Compression and tension).

Bond stresses need not be investigated where post-tensioned and untensioned steels are distributed nearly uniformly over the tensile area and the diameter of the mild steel bars does not exceed 1 in., and provided that the beam has sufficient anchorage. This rule also applies to members compressed with cables or stranded wires provided that the applied tensile force is less than 30 tons. For the design load the calculated bond stress is limited to the values in line 20 of *Table* III and for calculated breaking load by the values in line 21.

ANCHORAGE.—The force at the anchorage is assumed to be $\frac{1}{1{\cdot}75}$ times the breaking load of the member. In the case of pre-tensioned steel, the factor of safety of the bond must be ascertained by tests.

In the case of slipping between the steel and the anchorage, the steel may be further stretched, provided that the permissible stresses given in lines 22 and 23 of *Table* III are not exceeded by 5 per cent. in temporary over-stressing at the anchorages.

Thickened Slabs in Place of Beams.

THE Guelph General Hospital (*Fig.* 1), which has recently been completed in Toronto, has five stories and a basement, with a two-story penthouse on the roof containing machinery for the lifts and ventilation system. The building is in the shape of a letter L, one wing being about 208 ft. long by 46 ft. 6 in. wide and the other 102 ft. long by 48 ft. wide. The basement extends under the whole of the ground floor and projects 25 ft. 6 in. from the rear of the longer wing. The height of each story, floor to floor, is 11 ft. The structure comprises reinforced concrete slabs and columns with beams around the perimeter of the building only. The external walls are of brick and the internal partitions of hollow clay tiles.

This is one of the first buildings in Canada with wide bands of thickened slabs in place of beams. A plan of one of these beamless floors is shown in *Fig.* 2 and a cross section in *Fig.* 3. It is stated that the advantages of this type of floor compared with beams and slabs are that the total thickness is reduced, that there is greater freedom for the arrangement of equipment, that fewer columns are required, and that the absence of beams allows a more flexible arrangement of internal walls. Large openings through the floors are generally framed with beams, although not necessarily on all sides, and small openings may be made almost anywhere.

Each floor comprises a slab continuous over three spans across each wing; the central part, 10 in. thick, also spans longitudinally between strips with extra reinforcement between the two interior columns. The two outer spans are supported along the sides of the building by spandrel beams carried on columns. In the shorter wing these spans are 14 ft. 8½ in. and the slab is 5¼ in. thick; in the longer wing the spans are 12 ft. 10 in. and the thickness 4¾ in. The increased thickness of the outer slabs near the columns was allowed for in calculating the bending moments.

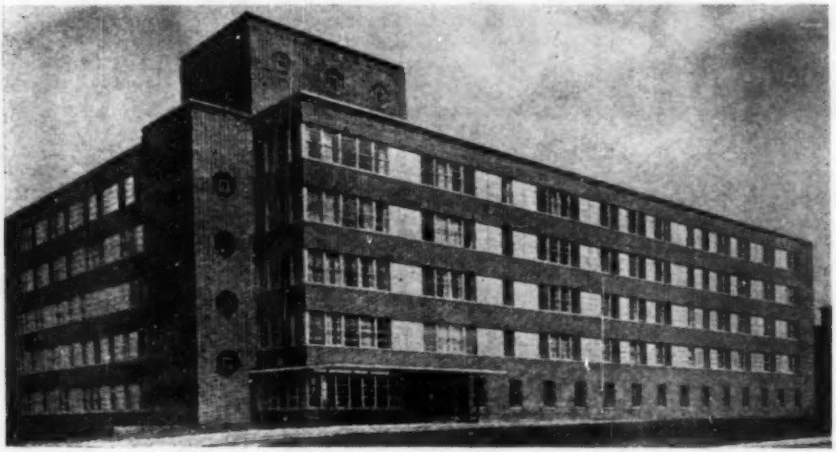


Fig. 1.—A Hospital with Beamless Floors.

Arrangement of Reinforcement.

The main reinforcement in the outer slabs comprises ½-in. bars at 6-in. centres, alternate bars being bent upwards into the central slab. In addition, in a lateral direction between the columns, there are at the top of the slabs three ½-in. bars at 9-in. centres, and at the bottom two ½-in. bars at 6-in. centres. These slabs are reinforced longitudinally with ½-in. bars at 1 ft. 4 ni. centres to resist the effects

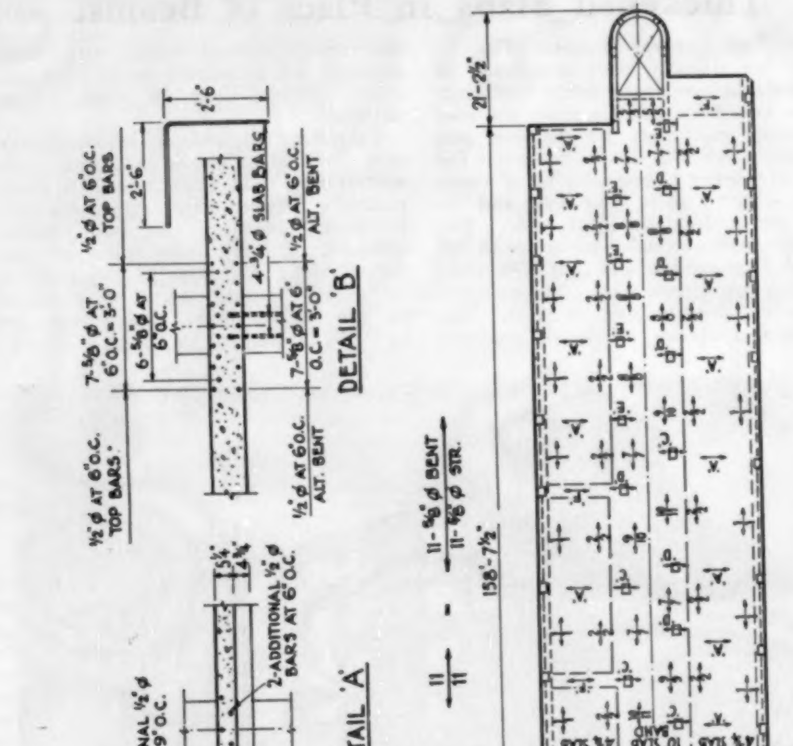


Fig. 2.—Plan of a Floor (The drawings do not show all the reinforcement).

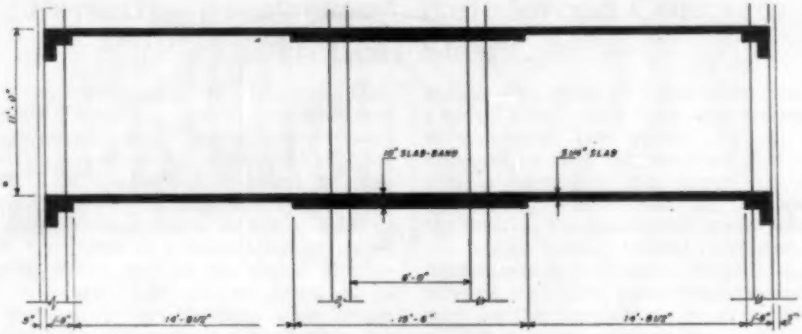


Fig. 3.—Cross Section through Floors of Shorter Wing.

of shrinkage and changes of temperature, and with two ⅝-in. bars adjacent to the spandrel beams and four ⅝-in. bars near the central slab. The central slab generally has ½-in. bars laterally at 1-ft. centres in the top and bottom, lapping with the bars extending from the outer slabs, and ⅝-in. bars at 6-in. centres in the top and bottom for a width of 1 ft. 6 in. on each side of the centre of the columns. The longitudinal reinforcement consists generally of 44 bars of ⅝ in. diameter, alternate bars being bent up over the lateral strips previously mentioned. The reinforcement of a floor is shown in *Fig.* 2. The specified compressive strength of the concrete at 28 days was 3000 lb. per square inch and the allowable stress in the steel 20,000 lb. per square inch.

The architects are Messrs. Marrani & Morris, the structural engineer Mr. W. S. Glynn, and the contractors the Pigott Construction Co. The foregoing description is taken from an article by Mr. Glynn in the February, 1953, number of "The Engineering Journal" (Canada).

"Concrete and Constructional Engineering" Prize Design.

THE proprietors of "Concrete and Constructional Engineering" offer each year a prize of £25 for competition among the students of reinforced concrete technology at the City and Guilds College of the Imperial College of Science and Technology, London. Last year the subjects from which the students could select were a railway bridge over the river Thames, underground oil tanks, a building of 25 stories, an exhibition hall with a "shell" roof, a jetty, a conference hall, a car-park with a shell roof, a laboratory, an auditorium, a science centre, and an office building of seven stories. There were twenty entries. The prize was awarded to Mr. B. W. Cooper for his designs for a railway bridge and underground oil tanks. The subjects were set by Professor A. L. L. Baker, Professor of Concrete Technology, and the assessor was Mr. F. E. Wentworth-Sheilds, O.B.E., P.P.Inst.C.E. In his report the assessor states that the designs show that the students had mastered modern ideas in reinforced and prestressed concrete design and construction.

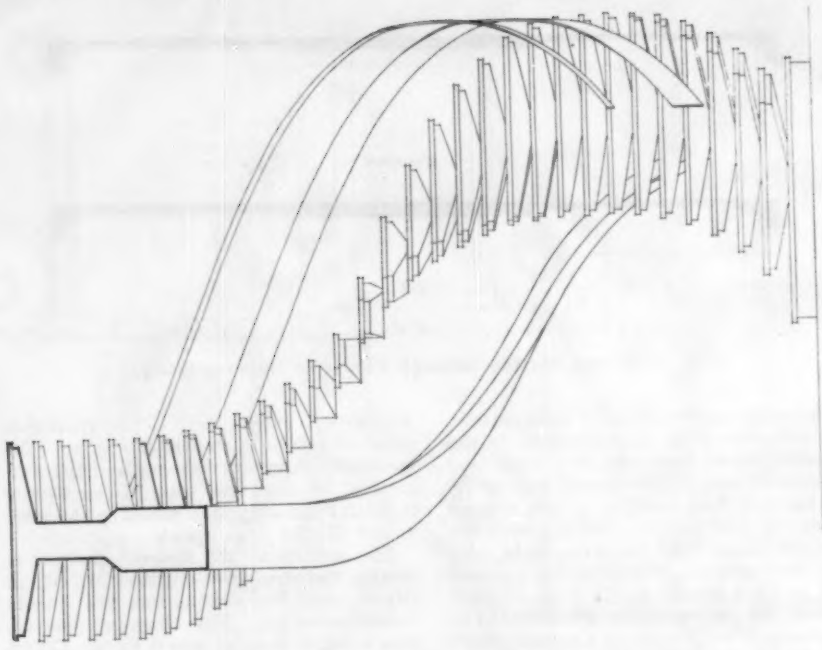


Fig. 3.—Front Elevation.

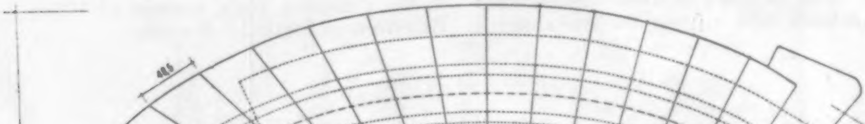


Fig. 2.—Plan.

Partially-prestressed Reinforced Concrete Helical Stairs.

RECENT extensions to Bologna University, for the Faculty of Chemical Agriculture, have partially-prestressed reinforced concrete stairs which are helical in shape and eliptical in plan (*Fig.* 1). The arrangement consists of a single curved beam which is fixed at the lower end in a foundation of trapezoidal shape and at the upper landing in a beam. The curved beam is of I section, the marble-faced stair treads being extended on each side to form the top flange. The height between floors is 19 ft. 8 in. and the width of the treads is about 4 ft. The width of all the treads is the same over the beam, but varies from one side to the other because the nosings are normal to the elliptical outline in plan. A plan is shown in *Fig.* 2 and an elevation in *Fig.* 3 (see page 196).

The superload was assumed to be 87½ lb. per square foot and the maximum concrete stress 1000 lb. per square inch. The main beam is prestressed by two cables, each consisting of twelve high-tensile steel wires of 5 mm. diameter, in a metal tube; each cable was tensioned to 22 tons. The cables (*Fig.* 4) are arranged so that the resultant compression is central at the upper end and within the middle-third at mid-span; the load on the foundation is also within the middle third. Helical mild steel reinforcement is provided to resist torsional stresses, and mild steel reinforcement is used for the foundation and upper beam as shown in *Fig.* 4 (see page 198).

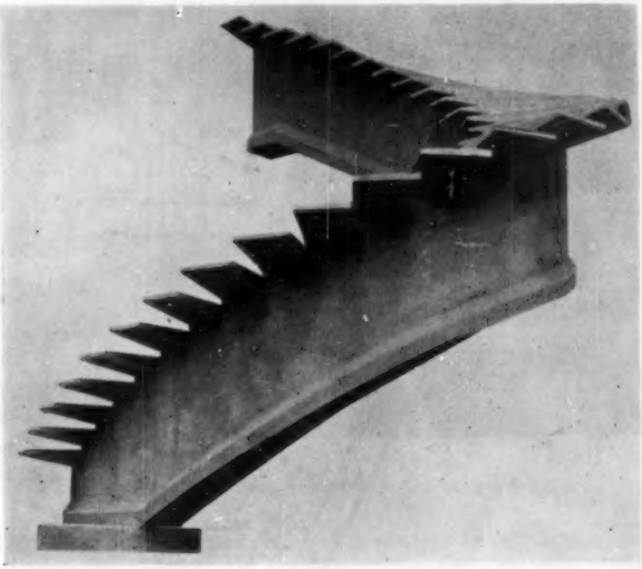


Fig. 1.

Under test loading, vertical deflections at mid-span of 0·45 mm. and 0·50 mm. on the internal and external faces were recorded and, assuming a modulus of elasticity of the concrete of 2×10^6 lb. per square inch, the stress was 284 lb. per square inch at the inner face and zero at the outer face. No permanent set remained after removal of the test load.

The architect was Sr. Luigi Riguzzi and the engineer Professor Giuseppe Rinaldi, chief engineer of the Corpo del Genio Civile of the Italian Ministry of Public Works.

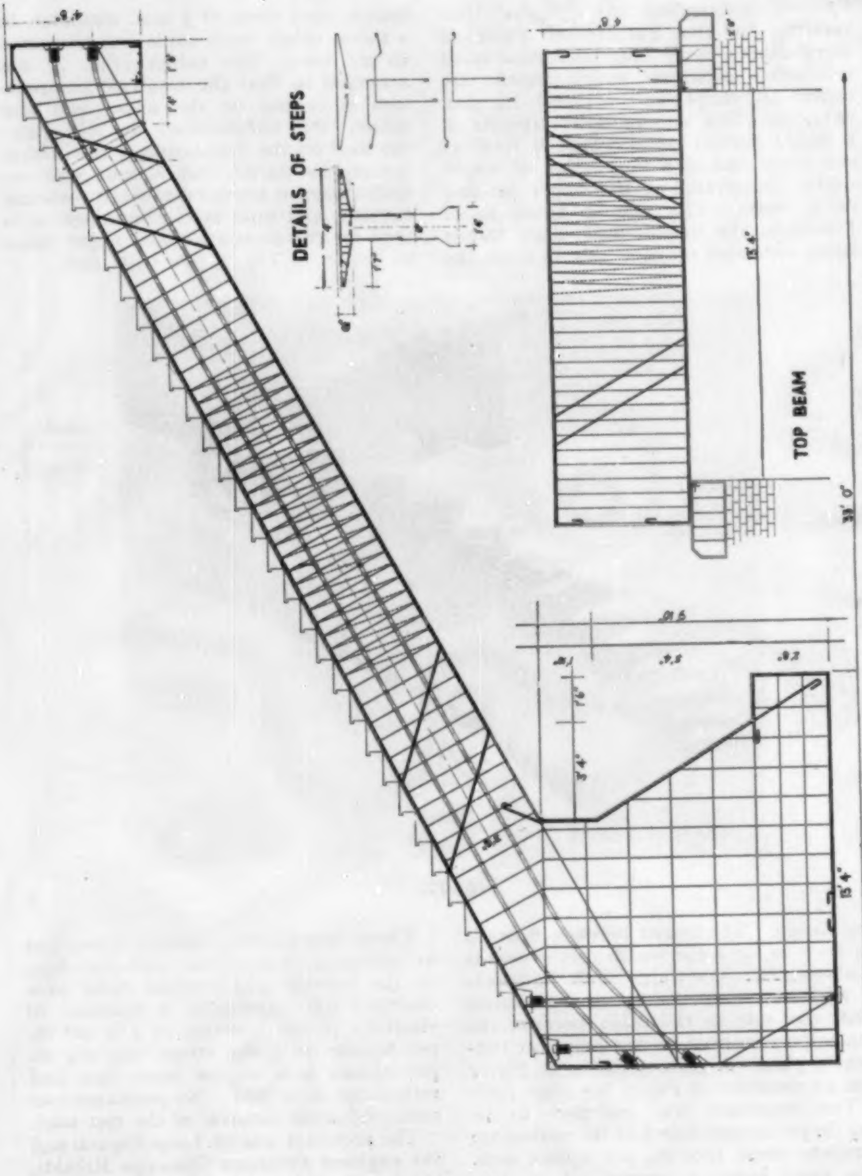


Fig. 4.—Details of Helical Stairs (*see page* 197).

Strengthening the Foundations of the Guildhall, London.

THE work of reconstruction of the Guildhall, London, included underpinning and strengthening the buttresses of the north wall. This work is of special interest as an ingenious application of a new process was used to solve an oft-recurring problem. The Guildhall is 500 years old, and the buttresses consist of stone in the upper portions and, in some cases, of a conglomerate of chalk flints and broken stone with lime mortar in the lowest portion. Each buttress weighs about 360 tons and will carry a load of about 60 tons from the arch rib in the roof. The new foundations are in a very restricted space and the hall is in constant use, and this, together with the friability of the materials at the base, the varying thicknesses and depths below ground of the walls, and the still different depths of the buttresses, presented an unusually difficult problem.

The method adopted was to sink bored piles 28 ft. long on each side of each buttress and penetrating up to 8 ft. into the blue clay below, to cast a reinforced concrete beam (*Fig.* 1) on these piles against each side of the buttress and slightly undercut, bore holes through the buttress in line with holes cast in the beams, and then insert high-tensile wires, tension them, and subsequently pressure-grout them and so squeeze the buttress and transfer the load to the piles. In the first buttress treated the piles were tested with a pressure of 50 tons on each pair of piles, which was a little in excess of the design load, and no pile settled more than about ⅛ in.

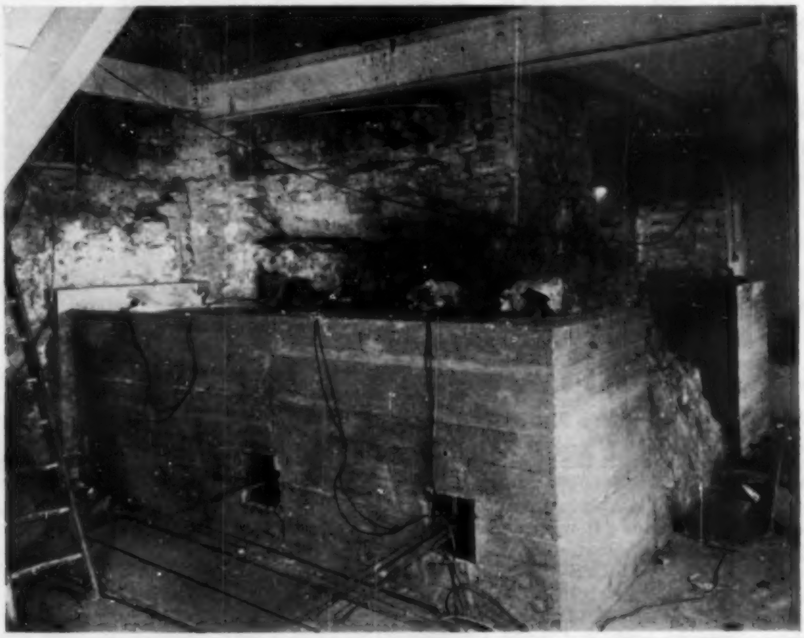


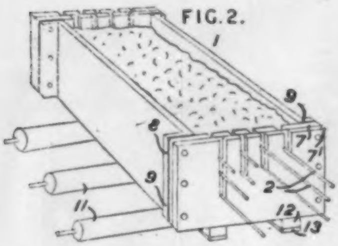
Fig. 1.—Method of Prestressing Buttresses.

The work is described in detail in a paper by Mr. Burnard Geen, M.Inst.C.E., the engineer for the work, in the Proceedings of the Institution of Civil Engineers, March, 1954, where the method is summarised as follows. (a) It was not necessary to disturb the existing foundations

by excavating below them ; (b) it was not necessary during the process seriously to weaken the buttresses ; (c) the supporting beams were placed at various levels to suit the varying levels of the existing foundations ; (d) when the operation on any buttress was completed it became immediately effective.

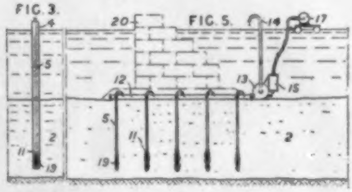
Patents Relating to Concrete.

Electrical Curing of Prestressed Concrete.



Apparatus for moulding prestressed concrete articles such as poles, railway sleepers, etc., in which the steel is heated by electric current to accelerate the hardening of the concrete comprises (a) one or more moulds (1) with end flanges (9) to which are bolted end walls consisting of double slotted plates (7, 7′), one or both end walls being of insulating material or insulation (8) being interposed between one or both end walls and the flanges (9) ; (b) anchor plates (3, 3′, 4, 4′), for holding the steel (2) after it has been tensioned ; and (c) means to insulate the moulds from their supports. The moulds may be on a roller track (11) and insulated therefrom by skids (12) strengthened with metal (13). The moulds may be arranged electrically in parallel paths or in series, as for example in *Fig.* 1 in which the leads (6, 6′) for the electric current are taken to the adjacent ends of the steel of two lines of moulds, the other ends of the steel being connected and the anchor-plates being insulated from earth and from each other.—No. 641,827. Metropolitan Vickers Electrical Co., Ltd. January 16, 1948.

Foundations.



In order that a bed of silt (2) may carry structures such as breakwaters (20) it is consolidated and stabilized by reducing its water and air content. Pipes (4) are driven into the bed at intervals and within these pipes are tubes (5) whose entry may be facilitated by pumping water down the tubes to remove any silt which may have entered the pipe. A filter bed (11) is formed around the lower end of each tube by passing a mixture of sand and water down the tube or the pipe, and the pipe is then removed. When pipes are removed the tubes are connected to a header (12) leading to a suction system comprising a submerged pump (13) which has an outflow (14) to the surface, whilst the air is removed since a chamber (15) on the pump (13) is connected to a vacuum pump (17) at water level. The lower end of each tube may be fitted with a screen (19) provided with a valve that permits the escape of sand but closes on the application of suction so that only air and water can be drawn into the tubes. Sand may be added to the silt before the silt is dewatered and de-aerated.—No. 652,570. S. Akerib. October 13, 1948.

[Publication of British patents has been delayed due to the war.]

Corrosion of Steel in Prestressed Concrete.

Mr. I. C. Parkins, B.Sc., Ph.D., has sent the following note on tests made by him on the resistance to corrosion of steel in prestressed concrete.

Details of the specimens used are given in *Fig.* 1(*a*). The concrete was a 1 : 1½ : 3 mixture of high-alumina cement, river sand, and river gravel, with a water-cement ratio of about 0·4. The steel was pre-tensioned and was left protruding for about 1 in. at each end, and the specimens, which were 9 ft. 6 in. long, were loaded as shown in *Fig.* 1(*b*). The load was applied by tightening the nuts on the central clamp and one of the end clamps, and pulling the other ends of the specimens together by means of a screw attached to a link which was calibrated, using electrical resistance strain-gauges. When the required load was attained, the nuts on the free end-clamp were tightened until the load was removed from the link, leaving the ends of the two beams pulling against each other. Sixteen beams were cast and loaded in this manner. Four pairs were loaded to working load, that is no tension in the bottom considering the specimens to be acting as beams; the other four pairs were loaded to slightly over one-and-a-half times working load, when visible cracks appeared in the bottom. Four pairs were stored in the laboratory and the others were placed between high and low water in the river Tay; one position was in fairly clean swift-running water, and the other was in more sluggish water which contained some sewage. The beams were left in these positions, alternately wetting and drying, for two years.

TABLE I.

Specimen No.	Cube strength* (lb. per sq. in.)	Test load (lb.)	Test condition	Load at failure (lb.)
1	6920	660	River	2320
2	,,	660	Laboratory	1980
3	7560	660	River	2240
4	,,	660	Laboratory	2150
5	8230	1000	Laboratory	2150
6	,,	1000	Laboratory	2320
7	8060	1000	River	2320
8	,,	1000	River	2500

* At age of 24 hours.

Twelve high-tensile steel wires of 0·079 in. diameter were used in each beam, and the final stress in the wires, after losses had occurred, was about 156,000 lb. per square inch. Therefore, with this wire, which had an ultimate stress of 310,000 lb. per square inch, corrosion had to penetrate to a depth of only just over one-hundredth of an inch for the stress in the wire to be increased to the ultimate stress.

Two pairs of specimens were destroyed when the part of the pier to which they were attached was damaged during a storm. The other specimens were subjected to a simple bending test to failure,

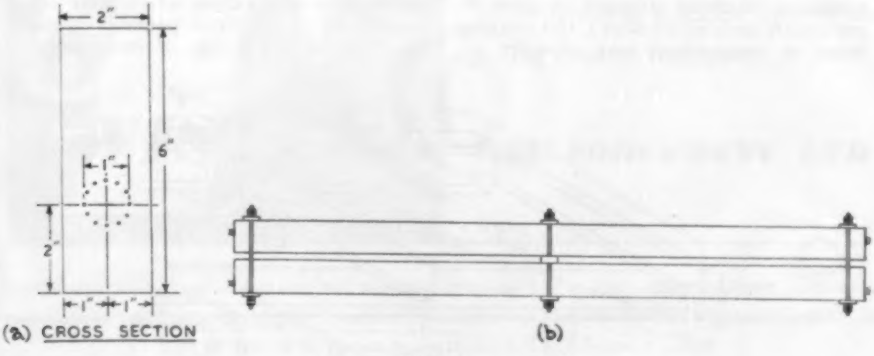


Fig. 1.

and the results are given in *Table* 1. It is seen that there was little difference in the ultimate strengths of the specimens, and none of the wires broke when the specimens failed. When the specimens were broken after testing the wires were still free from any sign of corrosion, although the wires that had protruded from the ends were completely eaten away.

This test suggests that prestressed concrete should be at least as resistant to corrosion as reinforced concrete. In particular, the tests indicate that in the case of pre-tensioned steel, the danger of corrosion penetrating along the wires from the exposed or poorly-protected ends is not great provided that the concrete is of good quality.

The author is indebted to Professor W. T. Marshall for his interest in and supervision of this test, and to the Dundee Harbour authorities for the facilities to carry it out.

Plastic Moulds for Precast Slabs.

For the construction of a Naval Air Station at Miramar, California, U.S.A., many precast slabs, mostly 24 ft. long by 4 ft. wide, have been used. The slabs are 1¼ in. thick for roofs and 1½ in. thick for floors. Along the sides are ribs 10 in. deep by 3½ in. wide at the bottom increasing to 5½ in. at the soffit of the slab; at 4-ft. centres there are transverse ribs 4½ in. deep by 2 in. wide. The parts of the slab between the transverse and longitudinal ribs are designed as separate slabs spanning in two directions for an imposed load of 75 lb. per square foot. The reinforcement in the slab consists of high-tensile steel mesh bent downwards into the longitudinal and end ribs. Each longitudinal rib contains one deformed bar of 1⅜ in. diameter near the bottom and a ½-in. bar near the top; each transverse rib contains one ½-in. deformed bar near the bottom. A floor slab is shown in *Fig.* 1. The concrete contained 750 lb. of cement per cubic yard and had a minimum crushing strength of 4000 lb. per square inch at 28 days; the working stress in compression was 1800 lb. per square inch. To enable the slabs to be handled the day after casting, a crushing strength at 24 hours of at least 800 lb. per square inch was required.

The parts of the mould (*Fig.* 2) to form the soffits of the slabs and the sides of the ribs were made of a plastic material reinforced with a fabric of woven glass fibres. Each former was about 4 ft. square, extending the full width of the slab and to the centres of adjacent transverse ribs; six such formers were required for a slab 24 ft. long. These formers were made from wooden patterns. At the centre of each former was a hole for the attachment of a device to facilitate its removal.

The formers were arranged on wooden bases to enable slabs 24 ft. long to be cast, the edges of the slabs being formed by metal sides hinged to the bases (*Fig.* 3). The bases were carried on wooden frames at a convenient height for working. To the underside of the base were attached water-pipes and valves which were connected, by ¼-in. pipes passing through the hole in the centre of each former,

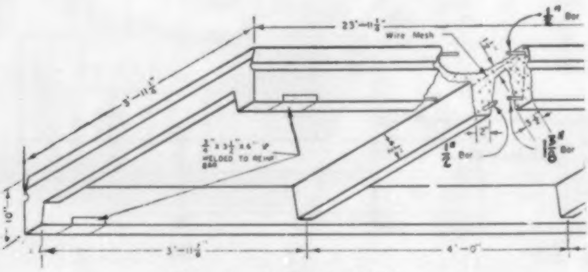
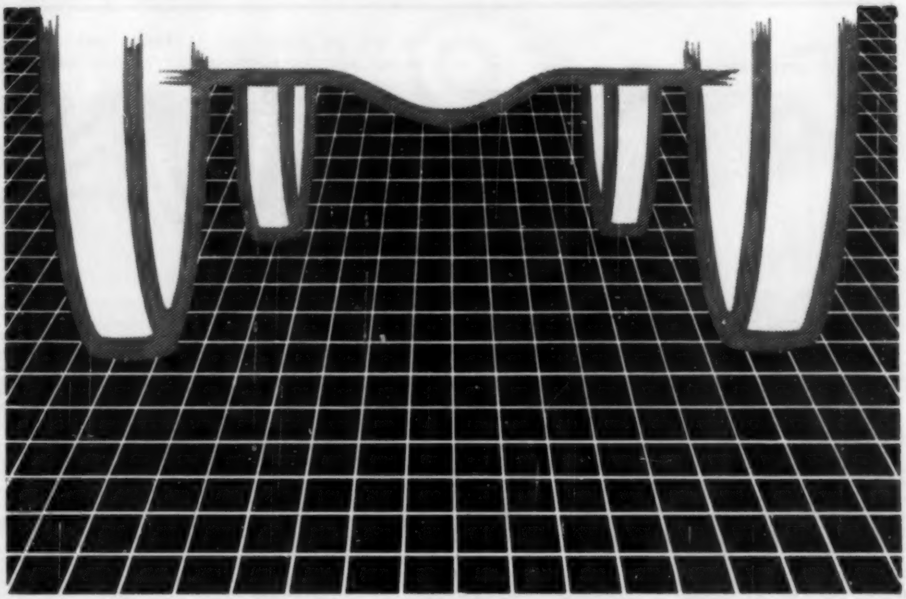


Fig. 1.—A Floor Slab.



to rubber sheets $\frac{1}{8}$ in. thick by 10 in. square set in depressions of the same size in the formers. The rubber sheets were fastened to the moulds by waterproof tape.

To strip a mould, water was introduced between the rubber sheet and the mould and pressure maintained until the slab was free. The water depressed the centre of the plastic former under the rubber sheet and escaped into the space between the former and the slab. After about five minutes water could be seen leaking from the edges of the moulds and the slab could then be lifted.

The concrete did not bond to the plastic material which, after removal of the slab, was cleaned by compressed air and wiped with a damp cloth. The metal sides were oiled. After fifty uses there was no sign of deterioration of the moulds.

The slabs were lifted from the moulds by a crane with a device as shown in *Fig.* 5 and stacked ten high. Erection was by crane using the device shown in *Fig.* 5 or the cradle shown in *Fig.* 4.

The building was designed by Messrs. Kistner, Curtis & Wright, and the work described was carried out by the Trepte Construction Co. The foregoing is abstracted from an article by Mr. M. R. Montgomery and Mr. T. G. Atkinson in the Journal of the American Concrete Institute for May, 1953.

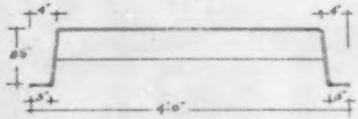


Fig. 2.—Section through Plastic Mould.

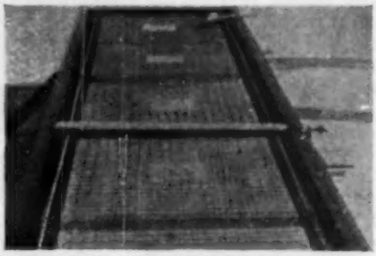


Fig. 3.—Mould for Slabs.

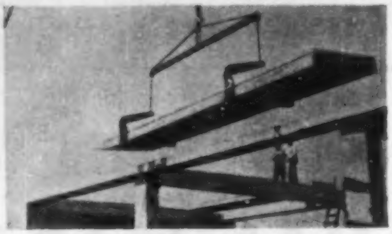


Fig. 4.—Cradle for Lifting Slabs.

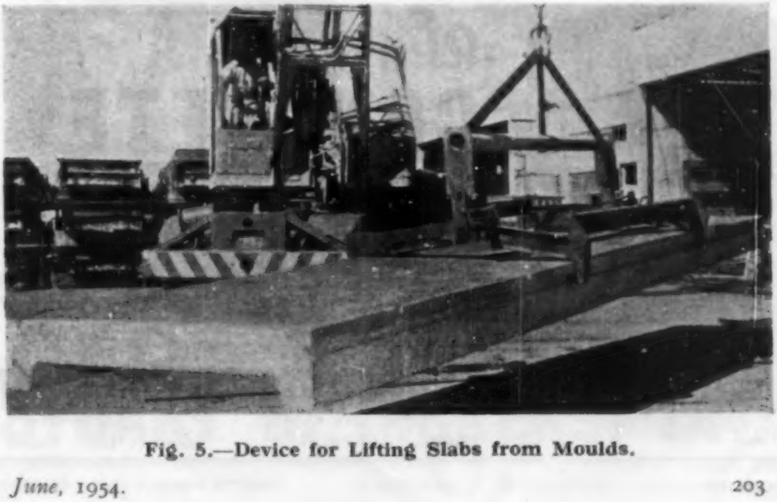


Fig. 5.—Device for Lifting Slabs from Moulds.

Earth Pressure on Tunnel Lining.

An investigation to determine the pressure on a tunnel in London clay has been carried out by the Building Research Station in conjunction with the Metropolitan Water Board and is described in the Report of the Building Research Board.

The tunnel is 9 ft. diameter, with its axis 90 ft. below ground level, and is to convey water under a head of about 140 ft. The lining consists of plain precast concrete segments, 6 in. thick, of which ten form a ring of the lining and are jacked in position against the clay by tunnel shield jacks.

In order to determine the pressure of the clay on the wall of the tunnel gauges were cast in nine of the rings and the pressures measured over a period of about a year. The gauges, consisting of steel cylinders, measured the forces by means of the change in frequency of a vibrated wire anchored axially to the ends of a cylinder. They were used in two ways, namely, (1) Set radially in the thickness of the concrete segments to bear on a floating steel plate placed flush with the external face of the concrete; twenty of the gauges were used in segments in three rings, and they measured the radial pressure of the clay on the lining. (2) Six pairs of gauges were placed circumferentially in split segments in four rings to measure the circumferential thrust.

As the segments were jacked into position the radial pressures and the circumferential thrusts rose immediately, then fell, and remained nearly constant. The radial pressures on the gauges around any one ring varied appreciably, but the average was about $2\frac{1}{2}$ tons per square foot, except in one ring near a substantial band of claystone rock where the average radial pressures were nearly 5 tons per square foot. The circumferential thrusts also corresponded to an average uniform

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SITUATION VACANT. Draughtsman. The Stanton Ironworks Co., Ltd., near Nottingham, has a vacancy on its prestressed concrete plant for a mechanical draughtsman to prepare drawings and carry out minor design calculations for moulds and tackle. Some experience of prestressed or reinforced concrete would be an advantage but is not essential. The post is progressive and superannuated. Write in confidence with full details of qualifications to the Staffing Director.

SITUATION VACANT. Civil engineers interested in road construction required. Appointments will be, initially, at the company's central laboratory, where training will be given in soil mechanics, asphalt and concrete technology. Engineering degree or equivalent desirable. Write, stating age, qualifications and experience, to George Wimpey & Co., Ltd., Central Laboratory, Lancaster Road, Southall, Middlesex.

SITUATION VACANT. Structural engineer required for research and development work in reinforced concrete construction. Applicants must have university degree in civil engineering, or other recognised civil or structural engineering qualifications, and experience in structural testing, including use of strain gauges. Progressive position either as career in laboratory or by later transfer to other departments of the Wimpey organisation. Write to George Wimpey & Co., Ltd., Central Laboratory, Lancaster Road, Southall, Middlesex.

SITUATIONS VACANT. Ministry of Works. Civil engineering assistants required for work on various sites. Candidates must be of British birth, and competent to check setting out with theodolite and level, or have knowledge of soil mechanics. Students who have passed Parts A and B of the A.M.I.C.E. examination will be considered, but, without this, some previous site experience is required. Although unestablished, these posts have long-term possibilities. Hostel accommodation available on sites. Salary range £420 to £640 per annum ; starting pay up to £550 according to age and experience. State age, details of training, and experience, to W.G.10/C.W.E.3(G), Ministry of Works, Abell House, John Islip Street, London, S.W.1.

(*Continued on page 205.*)

radial pressure of about $2\frac{1}{2}$ tons per square foot and suggested a variation in thrust around the rings. The axis of the tunnel was 90 ft. below ground level and the weight of the overlying saturated ground corresponded to a uniform radial pressure of about $5\frac{1}{2}$ tons per square foot, but water leaked freely into the tunnel and, if allowance were made for hydrostatic uplift, the radial pressure would be only about $2\frac{1}{2}$ tons per square foot.

Measurements were also made, using a screw micrometer stick, of the change in diameter of the lining on five diameters on each ring incorporating load-measuring instruments. These observations showed a steady vertical flattening of the tunnel lining amounting to a few hundredths of an inch, a slight increase in horizontal diameter, and a slight decrease in total circumference.

The Training of Supervisors.

THE City and Guilds of London Institute has prepared for the 1954–1955 session a one-year part-time course in concrete practice which is intended to provide a qualification for supervisors and potential supervisors engaged in concrete work. The course comprises 24 lectures. Full particulars may be obtained (price 6*d.*) from the City and Guilds of London Institute, Department of Technology, 31 Brechin Place, London, S.W.7. Similar courses are to be held at Croydon, Cardiff, and Edinburgh.

(Continued on page 206)

The Quality of Concrete.

A SYMPOSIUM on "Mix Design and Quality Control of Concrete" was arranged by the Cement and Concrete Association and held in London on May 11 to 13 last. The following is a list of the papers presented.

Design of high-strength concrete mixes, by H. C. Erntroy and B. W. Shacklock; The design of concrete mixes on the basis of flexural strength, by P. J. F. Wright; Mix design and abrasion resistance of concrete, by C. L. a'Court; Mix design for frost resistance, by A. R. Collins, M.B.E.; Design of concrete mixes for compaction by surface vibrators, by R. H. H. Kirkham; The effect of the vacuum process on concrete mix design, by D. F. Orchard; Steam curing and its effect upon mix design, by A. G. A. Saul; A classification of natural sands and its use in mix design, by A. J. Newman and D. C. Teychenné; The control of concrete quality: a review of the present position, by F. N. Sparkes; Method of achieving control of quality, by L. J. Murdock; Control of concrete quality on the small site, by J. H. Spanton; Quality control and its effect on structural design, by F. G. Thomas; Quality control for precast concrete, by F. Cornelius, M.C.; Economic factors in the choice of aggregate grading in relation to quality control, by D. A. Stewart, M.B.E.; Quality control for road and airfield construction, by J. M. Fisher; Quality control for dams and mass concrete, by W. Dick; Ready-mixed concrete: quality control refinements, by L. Boyd Mercer; Some problems involved in destructive and non-destructive testing of concrete, by R. Jones and P. F. J. Wright; The application of statistics to concrete quality, by F. R. Himsworth; The problems of specifying and standardizing practice for concrete mix design and quality control, by A. R. Collins, M.B.E.

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